Pore pressures in front of a slurry shield: development and decline

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Abstract: Two theories with analytical solutions are reported in the literature to describe the increase and decline of pore pressures in front of a TBM drilling in saturated sand. The first theory considers transient flow in a semi-confined aquifer with elastic storage, while the second one assumes different conditions of unconfined steady-state flow governed by the filter cake at the tunnel face. This paper determines which theory is more appropriate to model the results from the field measurements performed at the Green Hart Tunnel in the Netherlands. Both models are tested at different positions around the tunnel, during both drilling and standstill, and the results are discussed.

1 INTRODUCTION

Mechanized tunneling with a slurry shield has been widely used for tunnel projects in saturated sandy soil. The experience on shield tunneling, however, started in the Netherlands only two decades ago [1]. Because of the limited knowledge of shield tunneling in permeable saturated sandy soil, a series of in-situ measurements and studies have been carried out during the construction of the first projects in the Netherlands. One of the findings of that research was that there is an excess pore pressure present in the soil in front of the TBM during drilling, which influences the stability of the tunnel face [2-4]. This excess pore pressure disappears during standstill (Fig. 1a). In Netherlands, excess pore pressures were measured during the tunnel driving at various projects, such as the Second Heinenoord Tunnel, the Botlek Rail Tunnel, the Green Hart Tunnel (GHT) and the South/North Line in Amsterdam [4-7].

Broere [3] proposed a method to predict these excess pore pressures assuming that drilling takes place in a semi-confined aquifer and that equilibrium is not achieved immediately, so that transient conditions must be considered. The development of the excess pore pressure when drilling starts, and the decline of these pressures during standstill, are governed by the elastic storage in the sand. The excavation is modelled thorough its discharge, which is estimated considering that the amount of water displaced by the infiltrating slurry is roughly equal to the porosity of the excavated material. Bezuijen et al. [4 and 8] proposed a model where the pore pressure variations are governed by the properties of the filter cake, and the excavation is set in an unconfined aquifer where steady-state conditions can be considered.

The dissipation of the hydraulic head through the cake defines the piezometric head in front of the tunnel, which sets the new pore pressure distribution based on an assumption of radial flow.

The results from these two methods will be compared with the pore pressures measured around the construction of the GHT in the Netherlands. The location of eight pore pressure transducers (PPTs) around the tunnel cross-section can be seen in Fig. 1b. The two instruments further away from the TBM are just marked at their depth (WR1, WR2). All instruments were placed in the ground along the tunnel chainage 4219 m, with the exception of WB0 (4221 m) and WC0 (4223 m). This layout allows the pore pressures to be evaluated both in time and space.

The records of the shield position are only available after the face of the TBM had passed the instrumentations section. However, other cases [7,9] reveal that the increments of pore pressure are more or less symmetric regarding the distance between the face and the instruments. Therefore, the distances after the section can be considered equivalent to distances before the section.

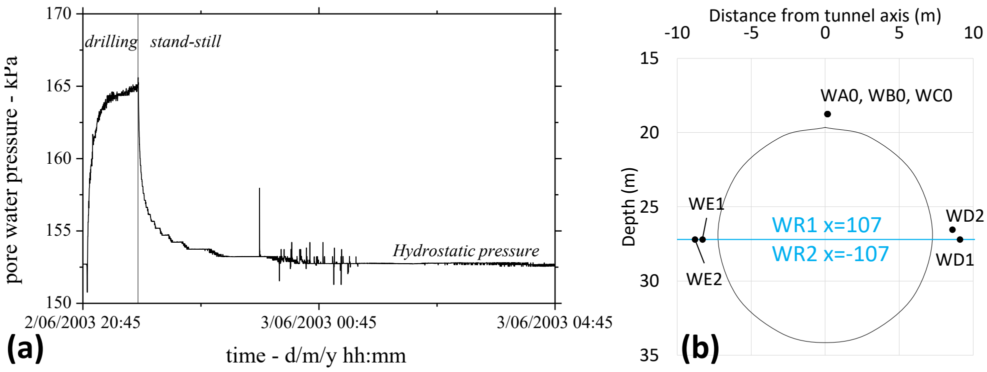

Figure 1. Pore water pressure during the process of tunneling at Green Hart Tunnel: drilling and standstill (a), Sketch of pore pressure transducers at ring 2117 at GHT (b).

2 Transient Equations

Broere [3] proposed time-dependent formulas for the build-up and dissipation of excess pore pressures. While the TBM is drilling, the excess pore pressure is given by:

$$\varphi - \varphi_0 = \frac{Q\lambda}{4kH}\left[erfc\left(\frac{xu}{2\sqrt{t}} + \frac{\sqrt{t}}{u\lambda}\right)\exp\left(\frac{x}{\lambda}\right) - erfc\left(\frac{xu}{2\sqrt{t}} - \frac{\sqrt{t}}{u\lambda}\right)\exp\left(-\frac{x}{\lambda}\right)\right] \quad (1)$$

where the leakage length $\lambda = \sqrt{kH\tilde{c}}$, with $\tilde{c}$ the hydraulic resistance of the confining layer; k the permeability of aquifer; H the thickness of the aquifer; $u = \sqrt{S_s/k}$, with S_s the coefficient of specific storage; t the time.

This equation is valid when the slurry penetration velocity is higher than the drilling velocity. For the GHT that is not the case. Therefore, the real excess water pressures will be lower than the values calculated by Eq. (1), and the section $Q\lambda/4kH$ in Eq. (1) should be replaced by $\Delta p_p/2\gamma_w$.

During the standstill, the decreasing excess pore pressure is calculated by:

$$\varphi - \varphi_0 = \frac{\Delta p_p}{2\gamma_w}\left[erfc\left(\frac{xu}{2\sqrt{t}} + \frac{\sqrt{t}}{u\lambda}\right)\exp\left(\frac{x}{\lambda}\right) + erfc\left(-\frac{xu}{2\sqrt{t}} + \frac{\sqrt{t}}{u\lambda}\right)\exp\left(-\frac{x}{\lambda}\right)\right] \tag{2}$$

where Δp_p is the remaining excess pore pressure; and γ_w is the specific weight of water.

At the GHT, the retaining excess pore pressure Δp_p was estimated as 22 kPa from the available field data. The permeability of the aquifer is about 4×10^{-4} m/s [9], and the thickness is $H = 35$ m. As the ground conditions are similar to the Botlek Rail Tunnel, the coefficient of specific storage S_s is assumed 7×10^{-4} m^{-1} [6]. Combining the results from Eq. (1), in its modified form, at $t = 0$ and the measured pore pressure as a function of the distance in front of the TBM, the leakage length λ can be estimated as 50 m. Fig. 2 shows the pore water pressure measured in the transducer WA0 (see Fig. 1 for its position). The TBM passed the instrument section at $x = 4204$ m. This shows that the maximum pore pressures can be predicted well.

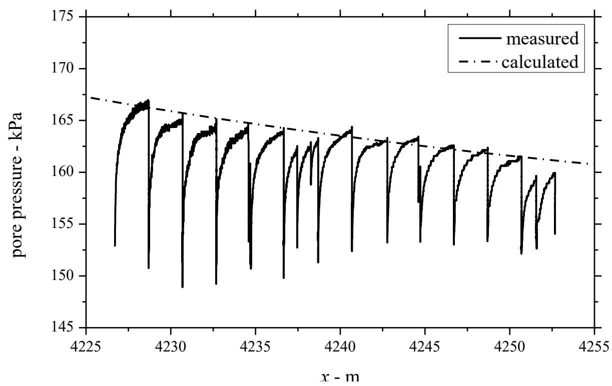

Figure 2. Pore pressure in front of slurry shield as a function of the distance from the TBM front at GHT compared with calculated maxima assuming λ=50m.

3 Steady State Model

This method assumes a radial flow from the tunnel and takes into consideration the permeability of both the slurry and the water in the soil, and the radius of the tunnel. The influence of slurry penetration and the filter cake conditions on the pore pressure is evaluated as well. Moreover, the parameters for this model can be determined from laboratory test on slurry infiltration.

The formula to calculate the maximum pore water pressure in front of a TBM, previously described by Bezuijen [4], is:

$$\phi = \phi_0 \left[\sqrt{1 + \left(\frac{x}{R}\right)^2} - \frac{x}{R} \right] \tag{3}$$

where ϕ_0 is the piezometric head at the tunnel face, ϕ the piezometric head at a distance x in front of the tunnel face, and R the radius of the tunnel.

The formula is valid for situations where the permeability of soil around the tunnel is constant. The hydraulic gradient for $x = 0$ can be obtained by differentiating Eq. (3) for x. The filter velocity can be calculated with Darcy's law, from where the pore velocity (v_p) can be calculated as:

$$v_p = \frac{k\phi_0}{nR} \tag{4}$$

3.1 Pore water pressure decline after drilling

During standstill, for example because of ring building, the penetration of slurry in the soil will continue until the maximum penetration depth is reached. Assume the maximum penetration depth is L and that the piezometric head is zero far from the tunnel. In that case Eq. (3) presents the relation between the piezometric head in the soil just in front of the slurry and the piezometric head in the mixing chamber can be written as:

$$\varphi_{mx} = \varphi_0 + \frac{v_p}{k_g}x + \frac{\varphi_{mx}}{\mathrm{L}}x \tag{5}$$

where k_g is the permeability of the ground before the slurry.

Since $v_p = dx/dt$ this leads to:

$$\frac{dx}{dt} = \frac{\varphi_{mx}(1 - x/L)}{\frac{n}{k}R + \frac{n}{k_g}x} \tag{6}$$

Eq. (6) is a non-linear differential equation. It can be solved numerically starting at $x = 0$ if there is no slurry penetration during drilling, or starting with a certain value of x if there had been slurry penetration during drilling. When $dx/dt = v_p$ is determined, Eq. (4) can be used to determine ϕ_0 as a function of time.

3.2 Pore water pressure development during drilling

The same principle can be used to estimate the increase in excess pore water pressure when drilling starts. Again Eq. (6) can be used, but it should be realized that during drilling the cutting wheel takes some soil in which the slurry is penetrated. And therefore to calculate x at various time steps the reduction x due to drilling has to be included:

$$x_{i+1} = x_i + \left(\frac{dx}{dt} - v_{TBM}\right).\Delta t \quad (7)$$

For this analytical method, because the calculation of x_{i+1} starts after the first standstill, the pore pressure is also calculated from the first period of standstill. For the first time step, this changes Eq. (7) to:

$$x_{i+1} = L + \left(\frac{dx}{dt} - v_{TBM}\right).\Delta t \quad (8)$$

where the suffixes i and $i+1$ refer to different times in a numerical solution and Δt is the time step between them.

4 DISCUSSIONS

The pore water pressures measured during drilling and stand-still are compared with the values calculated based on the theories from Broere (Section 2) and Bezuijen (Section 3).

Starting with the pressure drop at the end of drilling, the transient model is applied with the same parameters calibrated in Section 2. For the steady state model, the procedure from Section 3.1 is used, assuming the following parameters: $k_g = 2.10^{-5}$ m/s, $k = 4.10^{-4}$ m/s, $L = 0.07$ m extrapolated from laboratory tests [10], $\phi_{mx} = 4.16$ m, $n = 0.35$, and $R = 7.25$ m. Fig. 3a shows the comparison of measured and calculated results when the TBM stops at about 26 m from the PPT WA0. It can be seen that the pressures predicted by both theories fit the measured values well, but Broere's theory underestimate the excess pore pressure after about 500 s.

To simulate the pressure increase during drilling, Eq. (7) can be used with the parameters listed above. Figure 3b shows the results of both theories compared with the measured pressure increase. Both theories predict the excess pore pressure well, but again Bezuijen's theory shows a more reasonable agreement.

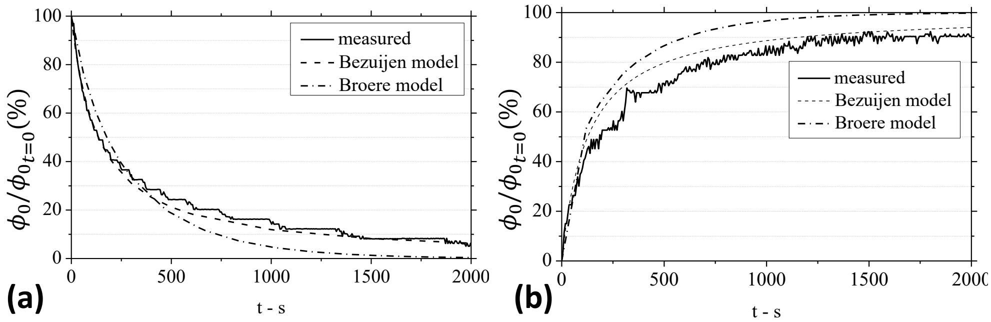

Figure 3. Pore pressure decrease when drilling stops due to slurry penetration at GHT (a) Pore pressure increase when drilling starts at GHT (b).

The pore pressure in sensor WB0 at ring 2118, located 2 m in front of the PPT WA0, during the course of three days, was plotted in Fig. 4. More than 23 cycles of pore water pressure increase and decline can be seen. The range of the cycles decreased from about 20 kPa to less than 10 kPa during the third day.

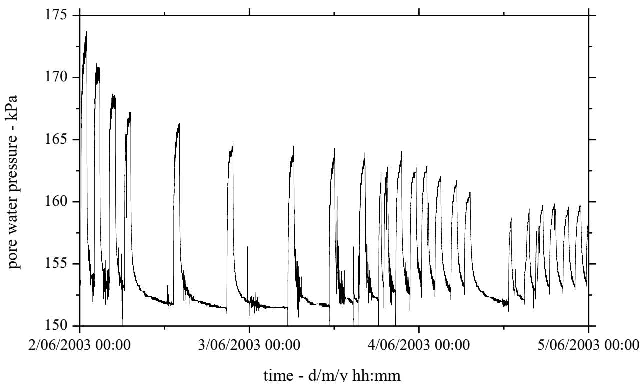

Figure 4. Pore pressure in WB0 with time at GHT.

To explore this mechanism in more detail, four cycles were analyzed, reflecting different distances between the TBM face and the instrumentation section. The period of these cycles were: 2/6 - 13:16 to 20:49; 3/6 - 05:27 to 11:17; 3/6 - 22:44 to 00:21 and 4/6 - 06:24 to 12:40, during which the TBM was drilling around the chainages of: 4229, 4231, 4241, and 4249 m, respectively. It is worth to recall that PPT WB0 was installed at 4221 m. For each of the cycles the magnitude of the water pressures was different. Therefore, the values are scaled between 0-100% for the range between the minimum and the maximum pressures at each cycle.

The resultant curves for the pressure development can be seen in Fig. 5a. For all the relative positions of the TBM face, the curves closely resemble one another. This was expected according to Bezuijen's model because the effect of distance from the TBM front is eliminated with this normalization. However, the transient model from Broere predicts that the excess pore water pressures, when drilling starts, depend on the distance x (see modified Eq. (1)). Fig. 5b shows the predicted pressure development according to Eq. (1), calculated with the parameters listed in Section 2. The pressures at a larger distance increased at a lower rate than those at shorter distances, but the difference was quite small.

The corresponding curves for the pressure decline are plotted in Fig. 5c. For all the relative positions of the TBM face, the curves closely resemble one another, with the exception of $x = 4229$ m. As the figure shows, the dissipation at this position was slower. According to the transient model from Broere, the course of the excess pore water pressures during standstill, also depends on the distance x (see Eq. (2)). Fig. 5d shows the calculated pressure decline according to Eq. (2) with parameters listed in Section 2. Similar to the curves for pressure development, when the distance was higher, the dissipation was slower.

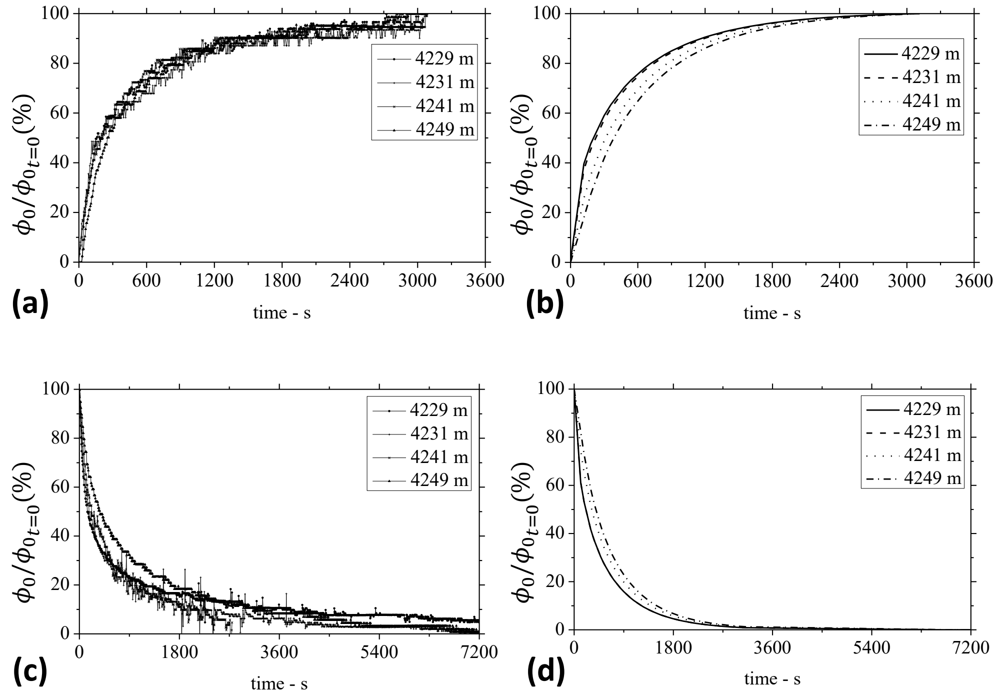

Figure 5. Pore pressure at different time in front of slurry shield at GHT. Measured pore water pressure increase in WB0 (a), Calculated pore water pressure increase in WB0 according to Broere's model (b). Measured pore water pressure decrease in WB0. (c) Calculated pore water pressure decrease in WB0 according to Broere's model (d)

4 CONCLUSIONS

Two analytical methods are presented to show whether or not excess pore pressures will be present in front of a TBM when drilling in saturated sand. The models also describe the pore pressure decrease when drilling stops and the increase in pore pressure when drilling starts. The following conclusions are possible from the measurements and the theory presented in this paper:

- The analytical theories developed by Broere and Bezuijen allow the increase in pore pressure over time, when drilling starts, as well as the decrease when drilling stops to be described. Bezuijen's theory seems to predict the course of excess pore pressure in front of a TBM better, as it takes into account the effects of slurry penetration and filter cake.
- Bezuijen assumes a homogeneous subsoil of sand with a constant permeability. That is not always the case and was not the case at the GHT. If there is a semi-confined aquifer in the subsoil, this has to be taken into account. At the GHT the excess pore water pressures were measured up to 100 m from the tunnel face, due to the long leakage factor

of the aquifer. In such a situation Broere's model can be used to calculate the maximum pressure and Bezuijen's for the course of the pressure when drilling starts or stops.

- From the excess pore water pressures measured at different positions after passing of the TBM, it could be determined that the elastic storage in the aquifer is not as large as follows from Broere's model and that the filtration layer between the slurry and the bentonite plays an important role for the excess pore water pressures.

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